SUSPENSION BRIDGE
OVER THE
LA GRASSE RIVER MASSENA CENTER, N. Y.

BY
J. L. STEWART
O. C. BADGER

ARMOUR INSTITUTE OF TECHNOLOGY

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DESIGN

Of A Highway Suspension Bridge Across The La Grasse River At Massena Center, St. Lawrence County, N. Y.

A THESIS

Presented By

John L. Stewart. ----------Orville C. Badger.

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Approved:

Alfred E. Phillips
Prof. Civil Engineering.

H. M. Raymond
Dean of Eng. Studies

L. C. Monin
Dean of Cult. Studies.

S U S P E N S I O N B R I D G E S .

Suspension Bridges may be classed under two main heads:-

(a),- Those composed of a light platform suspended from a cable , the loads passing directly from the floor to the cable.

(b),- Those consisting of a roadway supported by a truss which is hung from the cable by means of hangers.

Structures of the first class are called Unstiffened Suspension Bridges. Because of their lack of rigidity, structures of this type are limited to short spans and light loads.

Structures of the second class are called Stiffened Suspension Bridges. The applied loads are taken up by means of the stiffening trusses and distributed to the cables by means of hangers. Due to the rigidity of the trusses heavy concentrations or symmetrical loads are distributed over the cable approximately as a uniform load, so that it does not vary greatly from its original shape. Stiffened suspension bridges can be constructed rigid enough to carry railway and heavy city traffic.

Such men as Joseph Mayer, Gustave Lindenthal, and George S. Morison, have from time to time published articles in the leading Engineering Magazines on"Suspension Bridges", and it is due to their efforts, that this type of bridge has come to be recognized as an economical structure for long spans, both for heavy railway traffic and light foot traffic.

SUSPENSION BRIDGE
Over
LA GRASSE RIVER
At
MASSENA CENTER, N. Y.

The site of a Suspension Bridge over the La Grasse River at Massena Center, N. Y.

The La Grasse River is a tributary of the St. Lawrence River and is navigable to a point about three miles above the site of the bridge. The river also serves as a tail race for the power plant of the St. Lawrence Power Co.

The War Department required a 35 foot clearance for a distance of at least 250 feet at ordinary high water during the season of navigation, which is at about Elev. 160.00.

The design of the bridge provides a 45 foot clearance for a distance of 250 feet in the middle of the channel. This was deemed enough to take care of all emergencies that might arise locally.

This bridge is designed for highway traffic..

The bridge consists of three spans, a central span between towers of 400feet, and two side spans of 100 feet each. The central span is divided into 36 equal panels, the end spans into 9 equal panels, all spans being suspended from two cables. The anchorage roadway of 40 feet on each end makes a total length of 680 feet excluding grade approaches.

SUB-STRUCTURE

The highest water level on record being at 183.5, this height was fixed as the top of the masonry piers. These piers are the tower piers and they run down to Elev. 154.66.

The cable anchorages which also form the approaches are built of concrete. The space between the two wings is not built of solid concrete, but is left open and filled in with earth and stones.

On the North side of the river, soundings revealed a continuous bed of coarse sand and gravel. On the South side a gravely hardpan was encountered.

SUPER-STRUCTURE.

Each of the two cables is made up of 7 bridge cable strands 1 1/2 inches diam. each. Each strand has an ultimate strength of 267800#, equivalent to a unit strength of a little more than 200000# per. sq. in. Their modulus of elasticity is about 20,000,000. The main span cables are cradled to a batter of about 1 5/32 inches per ft., and the plane of the land span cables is coincident with that of the corresponding main span cable. The versine of the main cables center to center of towers is 38 feet at the assumed normal temperature, at which there is no stress in the stiffening truss due to live load.

The main supporting columns are vertical and spaced 25 foot centers. Each column or leg is anchored to the corresponding masonry pier with 2 1/2 inch steel rods embedded 8 feet in the masonry.

The stiffening trusses are 16 feet on centers and are 8 foot 7 1-2 inches deep, back to back of chord angles.

LOADING AND STRESSES.

The estimated dead load was 780# per lin. ft. of bridge, and a live load of 51# per sq. ft. floor area, (Class "C" Specifications for Highway Bridges.), on a 14 foot roadway. The cables, cable fastenings, etc., were designed for a maximum uniform live load over the whole bridge at a minimum temperature which was assumed at 40 deg. F. The floor was designed for a load concentration equivalent to a 15 ton road roller.

The limits of maximum stresses in the towers and anchorage steel, including bending stresses in the towers due to temperature changes is about the same as that of a live and dead in the trusses. The maximum unit stress in the cables under the extreme full load on the entire bridge at minimum temperature is less than 49000# per sq. in. That of the suspenders, which are made of the same grade of steel wire, is only 17000# per sq. in.

The estimated cost of this bridge complete is about $42000.

Articles on the estimate and construction of a bridge for this site appeared in the "Engineering Record" Oct. 5 and Nov. 2, 1912.

SPECIFICATIONS.

Coopers Specifications Class "C" will govern this design.

(1) The loading used in designing the floor system was according to Coopers Class "C" Spec. A 15 ton road roller was also designed for.

(2) The live load to be used in designing the stiffening trusses will be the uniform live-load for spans of 200 feet or over, as given by Cooper in his Spec. for Highway Bridges.

(3) Temperature and wind stresses will be neglected if they, combined, amount to 30%, or less, of the combined dead and live load stresses in the chord members. This is according to Coopers.

(4) Steel for the cables will be according to the specifications of John A. Robling's Son's Co., Trenton N. J., steelto have an ultimate strength of 200,000# per sq. in. A factor of safety of 4 will be used.

(5) The floor of the bridge is to be of 3 inch long leaf yellow pine plank. Guard rails 4 x 6 inches are used.

DESIGN OF FLOOR SYSTEM.

The design of a floor system for all types of gridges is practically the same, so no lengthy discussion will be gone into in designing this floor system. It will be designed according to the best modern practice.

THE STIFFENING TRUSS

A three hinged stiffening truss will be designed, as this type is much more satisfactory and has a greater carrying capacity than two hinged trusses.

The trusses are placed 16'-0''center to center and are 8'-~~3 1/2~~ 6'' deep, back to back of chord angles.

The stresses in the truss were determined according to Johnson, Bryan and Turneaure's "Modern Framed Structures, Part II". According to their theory, the cable always remains a parabola with its vertex at the center of the span, there-fore the hanger stresses are uniform over the entire span. There is no stress in the truss due to the dead load of the bridge.

The end trusses were designed on the assumption that the cable was a parabola. This assumption is not exactly correct, but as it gives stresses that are on the side of safety this method was used.

Temperature stresses were calculated according to Merriman and Jacobys' "Higher Structures", page 159.

Lateral stresses due to wind were calculated according to Johnson, Bryan and Turneaure, "Modern Framed Structures, Part II" page 321.

All similiar members were made of the same section, as this is considered best practice in modern suspension bridge design.

SUSPENDER RODS.

The suspender rods, which transfer the loads from the truss to the cables were designed in accordance with the theory advanced in Johnson, Bryan and Turneaure's "Modern Framed Structures," Part II.

DESIGN OF CABLES.

After the trusses have been designed and the dead and live loads estimated, we can then proceed with the design of the cable. Specifications say that the cables must be designed for the maximum tension to which it will be subjected. This condition occurs under full dead and live load.

The maximum tension for one cable in this bridge is 730,000#, thus a cable area of 14.6 sq. in. is required. Seven strands of 39 wires each, of No.2 steel wire were used.

Clamps for connecting suspender rods were designed according to experience and precedent. they cannot be mathematically designed. All suspender rods are fitted with standard clevises.

There is a cable deflection which is due to temperature and live load. The sag increased by temperature and live load. In order to take care of this sag the maximum deflection was determined, and in order that the bridge would never drop below the horizontal, the trusses were given a camber of about 5 feet.

The cables were cradled as this gives a greater resistance against lateral wind pressure.

DESIGN OF TOWERS.

The towers were designed according to Johnson, Bryan, and Turneaure's "Modern Framed Structures, Part II".

They were designed with three panels, the top and bottom panels being X braced, and the middle panel thru which the driveway passes is portal braced.

For cable seating see detailed drawing.

CABLE ANCHORAGES.

The function of the cable anchorage is to provide weight enough to counter act the tension of the cables. The bottom and side walls of the anchorage are built of 1:3: 5 concrete, and the space between the side walls is filled with stones and earth. Thus a cheap and stable anchorage is obtained. See masonry drawing for details of the anchorage.

LOADS.

Floor system	.232	Kips/lin.	Ft.	Cable.	
Truss	.200	"	"	"	"
Suspender Rods	.005	"	"	"	"
Live Load	.480	"	"	"	"
Dead Load	.330	"	"	"	"
Cable	.050	"	"	"	"
Total	1.297	"	"	"	"

Computations for a

HIGHWAY SUSPENSION BRIDGE AT

MASSENA CENTER, N.Y.

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CHICAGO, ILL.

DESIGN OF PLANKING ETC.

Assume width between stringers: 2'-9"

$S = \frac{Mc}{I}$ $M = \frac{Pl}{4}$ $S = 1000$ #□" for yellow pine

$c = \frac{d}{2}$ $\therefore$ $d = \sqrt{\frac{3Pl}{2Sb}}$ Assume roller distributes weight over 2 planks each 12" wide. Then b=24"

and $d = \sqrt{\frac{3 \times 7500 \times 33}{2 \times 1000 \times 24}} = 3.94''$ Planking will be 3"x12" since the weight of roller is probably distributed over a larger area and Specifications art. 18. state that floor plank shall have a thickness in inches at least equal to the distance apart of beams.

Weight of floor.

Weight of yellow pine 3.5# per ft. board measure

Floor area = 600 x 14 = 8400 □'

$8400 \times 3.5 \times 3 = 88200$#

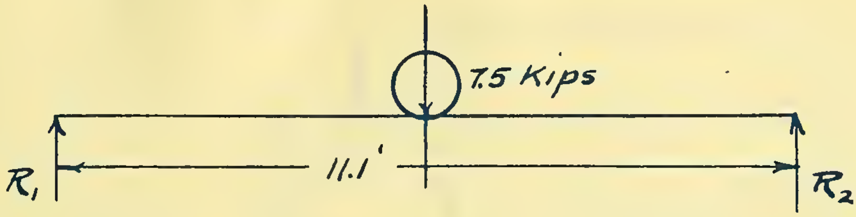
Wheel guards 6"x4" long leaf yellow pine

$600 \times 2 \times 3.5 \times .5 \times 4 = 8400$#

Total = 88200 + 8400 = 96600#

or 1790# per panel.

DESIGN OF STRINGER.



$R_1 = R_2 = 3.75$ Kips.

L.L. $M = 3.75 \times 5.55 \times 12 = 249.2$ Kip. in.

D.L. M. $= \frac{wl}{8}$ $w = 20^{\#}$ per ft. weight of

stringer $= 222^{\#}$ + dead load of floor on stringer

$= \frac{11.1 \times 33 \times 3.5}{4} = 320^{\#}$ Total $w = 542^{\#}$

D.L. $M = \frac{542 \times 11.1 \times 12}{8} = 9.0$ Kip. in.

Total $M = 249.2 + 9.0 = 258.2$ Kip. in. $=$

$258{,}200''^{\#}$. $S = 13000^{\#}\square''$ (art. 45 Spec.)

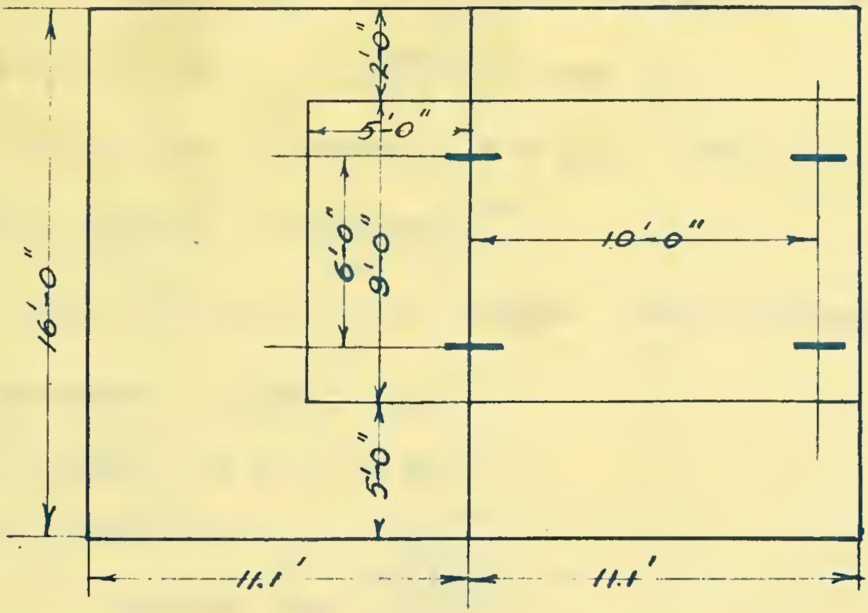
$\frac{M}{S} = \frac{I}{c} = \frac{258200}{13000} = 19.9$

From hand book $\frac{I}{c}$ for a $9''$ - $25^{\#}$ I beam

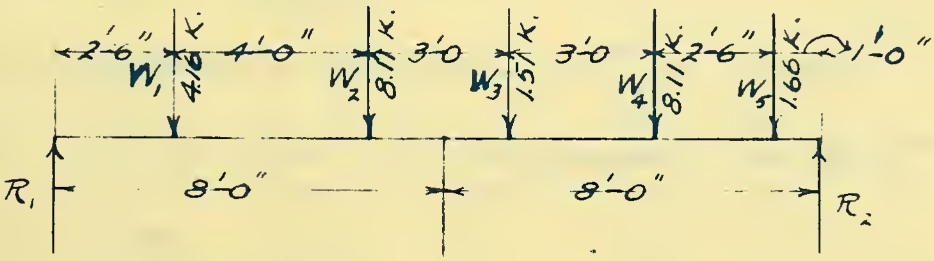
$= 20.4$ ∴ Use for all stringers $9''$ @ $25^{\#}$ per ft.

I beams.

DESIGN OF FLOORBEAM.



Concentrated load 15 tons on two axles 10' centers; and upon the remaining portion of the floor, a load of 100#□' of floor.



$$W_1 = \frac{5 \times 11.1 \times 100 \times 8.32}{11.1} = 4.16\ k.$$

$$W_2 = W_4 = 7.5 + \frac{7.5 \times 9}{11.1} = 8.11\ k.$$

$$W_3 = \frac{6.1 \times 100 \times 9 \times 3.05}{11.1} = 1.51\ k$$

$$W_5 = \frac{2 \times 11.1 \times 100 \times 8.32}{11.1} = 1.66\ k.$$

DESIGN OF FLOORBEAM (cont.)

Moments about R_2 $R_1 = 1.66 \times 1 + 8.11 \times 3.5 + 1.51 \times 6.5 + 8.11 \times$

$9.5 + 4.16 \times 13.5 \div 16 = \frac{173.13}{16} = 10.81$ K.

Mom. about W_2 $10.81 \times 6.5 = 4.16 \times 4$ Max. L.L. Mom.

$= 53.52$ kip. ft. $= 642240''^{\#}$

D.L. mom. figured from weight on floorbeam.

6 stringers $11.1' @ 25^{\#} = 1665^{\#}$

1 floor beam $16' @ 50^{\#} = 800^{\#}$

Floor, wheel guards $= 1790^{\#}$

$4255^{\#}$

details 10% $425.5^{\#}$

$4680.5^{\#}$

D.L. Mom. $= \frac{wl^2}{8} = \frac{4680 \times 16 \times 16}{8} = 112320''^{\#}$

Total Mom. $= 642240 + 112320 = 754560''^{\#}$

$\frac{I}{c} = \frac{M}{S} = \frac{754600}{13000} = 58$

From handbook $\frac{I}{c}$ for a $42^{\#}$ 15" I beam

$= 58.9$ ∴ Use for all floorbeams 15" @ $42^{\#}$

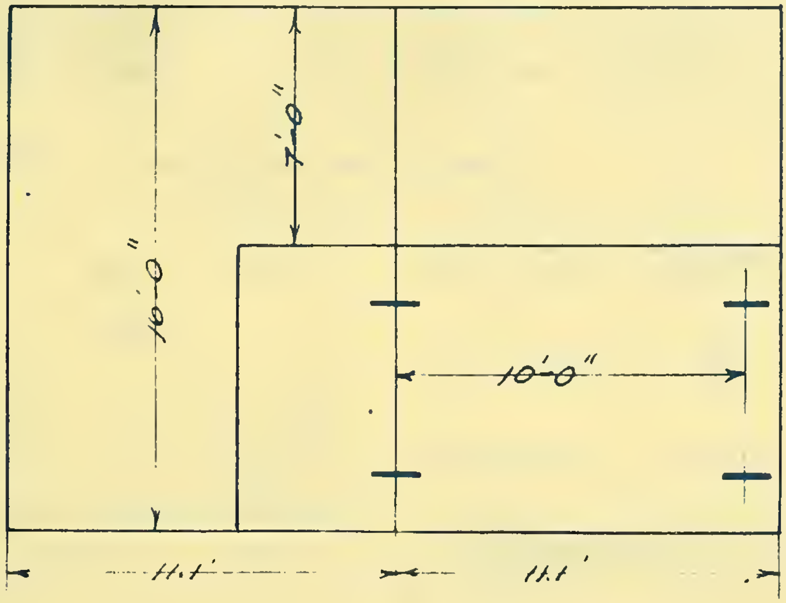
per ft. I beams.

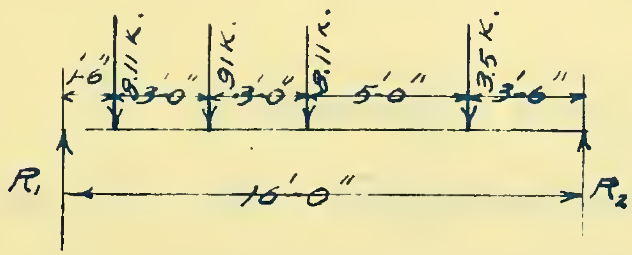
Weight of floor system.

$4600^{\#}$ per panel $= \frac{4.6}{11.1}$ or .42 kips per ft. of

bridge = .21 kips per ft. of truss.

MAX. END SHEAR ON FlOORBEAM (Live load)





Moments about R_2.

$R_1 \times 16 = 3.5 \times 3.5 + 8.11 \times 8.5 + .91 \times 11.5 + 8.11 \times 14.5$

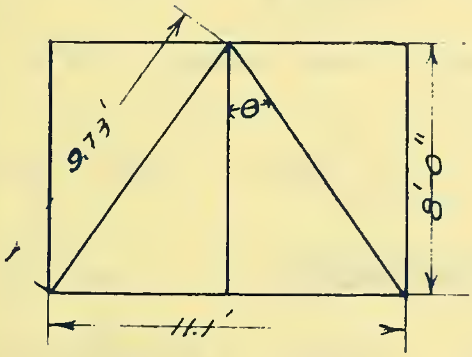
$R_1 = \frac{209.2}{16} = 13.1$ kips max. end shear.

Rivets in connection ∠'s to fl. beams

End shear = 13,100# Value of $\frac{3}{4}$" rivet in bearing $\frac{1}{2}$" pl. = 6750# $\quad \frac{13100}{6750} = 2$ rivets.

DESIGN OF STIFFENING TRUSSES.

Assumption made that trusses will not be stressed under dead load. L.L. = $60^{\#\square'}$ $w = 60 \times 1 \times 16 = .96$ Kips per ft. of bridge = .48 Kips per truss.



$$V = \frac{wl}{6} = \frac{.48 \times 400}{6} = 32.\text{ Kips}$$

$$\text{Max. Mom.} = \frac{wl^2}{53}$$

Merriman & Jacoby. Part IV.

$$\frac{.48 \times 400 \times 400}{53} = 1450 \text{ kip.ft.}$$

Sec θ = 1.21

For all diagonals. (main span)

Max. shear = 32 Kips. Designing stress = 1.21 × 32 = 37.5 Kips.

For all chords. (main span)

Designing moment = 1450 Kip. ft.

Design of Chord Section.

For tension allowed stress = $12500^{\#\square''}$

For compression $S = 12000 - \frac{55l}{r}$

Required area for tension $= \frac{1450}{8 \times 12.5} = 14.5^{\square''}$

Try 2 ∠'s $6'' \times 6'' \times \frac{13}{16}''$ area = $18.18^{\square''}$

2 rivets $\frac{1.42}{16.76}{}^{\square''}$ ∴ O.K.

DESIGN OF STIFFENING TRUSS (cont.)

For compression — $S = 12000 - \frac{55l}{r}$. r for 2∠'s $6'' \times 6'' \times \frac{13}{16}''$ $= 1.82''$ $l = 5.55 \times 12 = 66.6''$ $S = 12000 - \frac{55 \times 66.6}{1.82} =$ $10000^{\#\square''}$ $\therefore \frac{1450}{8 \times 10000} = 18.12^{\square''}$ req'd area.

∴ For upper and lower chords (main span) use 2∠s $6'' \times 6'' \times \frac{13}{16}''$.

Diagonals

Designing stress = 37.5 kips. For tension req'd. area $= \frac{37.5}{12.5} = 3.00^{\square''}$ Try 2∠'s $4'' \times 3'' \times \frac{1}{2}''$ area $= 6.5^{\square''}$, 1 rivet $\frac{.44}{6.06}^{\square''}$ O.K.

For compression $S = 10000 - \frac{45l}{r}$ $= 10000 - \frac{45 \times 12 \times 9.74}{1.25} = 5800^{\#\square''}$ Area req'd $= \frac{37.5}{5.8} =$ $6.45^{\square''}$ ∴ O.K. Use 2∠s $4'' \times 3'' \times \frac{1}{2}''$ for diagonals (main and end span). For end diagonals 2∠s $5'' \times 3'' \times \frac{1}{2}''$ will be used.

Verticals are used to make the unsupported length of chord smaller. 1∠ $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ will be used.

DESIGN OF STIFFENING TRUSS (cont.)

Chord Section (end spans)

Max. moment $= \frac{wl^2}{53} = \frac{1}{53} \times .48 \times \overline{200}^2 = 725$ kip ft.

For tension $\frac{725}{8 \times 12.5} = 7.25^{\square''}$ reqd. area. Try 2 ∠s $6'' \times 6'' \times \frac{7}{16}''$

area $= 10.12^{\square''}$
2 rivets $\underline{.76}$
$9.46^{\square''}$ O.K. For compression $- S = 12000 - \frac{55 l}{r}$

$= 12000 - \frac{55 \times 66.6}{1.87} = 10040^{\#\square''}$ reqd. area $= \frac{725}{8 \times 10.04} = 9.03^{\square''}$ O.K.

∴ Use for both chords 2 ∠s $6'' \times 6'' \times \frac{7}{16}''$

Weight of Truss.

Chords. — 4 ∠s. $6'' \times 6'' \times \frac{13}{16}''$ 11.1′ @ $31^{\#} = 4 \times 11.1 \times 31 = 1376.4^{\#}$

Diagonals — 2 ∠s $4'' \times 3'' \times \frac{1}{2}''$ 9.74′ @ $11.1^{\#} = 2 \times 2 \times 9.74 \times 11.1 = 432.4^{\#}$

Verticals — 1 ∠ $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ @ $4.1^{\#}$ 8′ $= 2 \times 8 \times 4.1 = 66^{\#}$

Total weight per panel = 1376.4 + 432.4 + 66 + 10 % for details $= 2062^{\#}$ or 2.1 kips $\frac{2.1}{11.1} = .19$ kips per ft. of truss.

DESIGN OF STIFFENING TRUSS (cont.)

Rivets in Truss. ($\frac{3}{4}$" rivets)

Shear = 10000 #□" Bearing = 18000 #□"

Single shear = 4418# double shear = 8836#

For diagonals - Stress = 6.5 x 5.8 = 37.7 kips (comp.)

$\frac{37.7}{6.75}$ = 6 rivets. Bearing on $\frac{1}{2}$" plate 6750#

For tension stress = 12.5 x 6.06 = 75.8 kips. $\frac{75.8}{6.75}$ =

12 rivets. Bearing on $\frac{13}{16}$" pl. = 11.2 kips.

For chords - For tension (main span) stress =

16.74 x 12.5 = 209 kips $\frac{209}{11.2}$ = 19 rivets. For comp.

stress = 18.18 x 10000 = 181.8 kips $\frac{181.8}{11.2}$ = 18 rivets

For tension (end span) stress = 9.46 x 12.5 = 108 kips

$\frac{108}{6.75}$ = 16 rivets required. For compression-stress

= 10.12 x 10.04 = 101.5 kips $\frac{101.5}{6.75}$ = 15 rivets

For verticals use 3 rivets.

DESIGN OF STIFFENING TRUSS (cont.)

Provision for expansion.

For a change of $\pm 70°F.$ from normal. Expansion coefficient for steel = .0000065

$L = etl = .0000065 \times 70 \times 400 \times 12 = \pm 2.18''$

$\therefore$ 5" will provide for total movement (main span)

For end span - $L = .0000065 \times 70 \times 100 \times 12 = \pm .545''$

$\therefore$ $1\frac{1}{2}''$ will provide for movement.

Hinges.

End hinges - End shear = 16 kips for one truss.

Pin area $= \frac{16}{10.0} = 1.6^{\square''}$ or a $1\frac{1}{2}''$ pin. A $2\frac{1}{2}''$ pin will be used.

Center hinge - Center shear = 6 kips per truss

Pin area $= \frac{6}{10} = .6^{\square''}$ or a 1" pin. A $2\frac{1}{2}''$ pin will be used for all pins.

DESIGN OF STIFFENING TRUSS. (cont.)

Temperature Stress for Truss.

$$S = \pm \frac{[9+24s^2]etEd}{10sl}$$ Merriman and Jacoby. Part IV.

S = temperature stress lbs. per sq. in.

s = sag ratio = .095

e = coefficient of expansion = .0000065" for 1° F.

t = range of temperature ± 70° F. from normal.

E = modulus of elasticity = 30,000,000 #□"

d = depth of truss = 8'-0"

l = length of span

$$\therefore\ S = \frac{(9+24\times\overline{.095}^2)\,.0000065\times70\times30{,}000{,}000\times8}{10\times.095\times400} = \pm 2640^{\#\square''}$$

DESIGN OF LATERAL SYSTEM.

Stress in Chords.

M = bending moment due to uniform wind load.

p = uniform wind load per unit length of bridge.

x = distance to section at which M is desired.

h = distance from ℄ of tower to ℄ of truss.

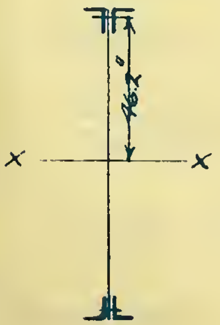
e = base to Naperian logarithms.

$$M = \frac{p}{c^2}\left[1 - \frac{e^{cx} + e^{(l-x)c}}{(e^{cl}+1)}\right]$$ (Johnson's Framed Struct.) Part II.

where $c^2 = \dfrac{f(H+H_w)}{hEI}$ f = sag of cable

H_w = due to wind load H = due to dead load

E = modulus of elasticity I = moment of inertia of truss. (I of chords about truss ℄.) x-x.



$I = (I_c + ad^2)2$ $d = 46.2''$ I_c = for 2 ∠s $6'' \times 6'' \times \frac{13}{16}''$ about center of gravity $= 2 \times 30.06$

$= 60.12$ $a = 18.18^{\square''}$

$$I = 2(60.12 + 18.18 \times \overline{46.2}^2) = 77730''^4$$

$$H + H_W = \frac{wl}{8S} = \frac{(.40+.33)\,400}{8 \times .095} = 385 \text{ Kips}$$

$$c^2 = \frac{f(H+H_w)}{hEI} = \frac{38\,[385]}{46 \times 30000 \times 77730} = .0000000597$$

$c = .00024$

M will be found for the point of max. L.L. M.

DESIGN OF LATERAL SYSTEM (cont.)

For M_a max. $x = .234\,l = .234(400) = 93.6' = 1123.2''$

Evaluation of quantities "e"

(e^{cx}) $\log e^{cx} = cx \log_{10} e = .00024 \times 1123.2 \times .434$

$= .11612$ log of 1.3065

$(e^{c(l-x)})$ $\log e^{c(l-x)} = c(l-x) \log_{10} e = .00024(3676.8)$

$(.434) = .38297$ log of 2.4164

(e^{cl}) $\log e^{cl} = cl \log_{10} e = .00024 \times 4800 \times .434 =$

$.49996$ log of 3.1620

w or $p = 300 + 30 = 330^{\#}$ Art. 39. Specifications

$$\therefore M = \frac{330}{.0000000597}\left[1 - \frac{1.3065 + 2.4164}{1 + 3.1620}\right] = 48{,}000{,}000''^{\#}$$

Lateral truss $= 16'\text{-}0''$ deep. Chord stress $= \dfrac{M}{d} =$

$$\frac{48{,}000{,}000}{16 \times 12} = 250{,}000^{\#} \qquad \frac{250000}{18.18} = 13700^{\#\square''}$$

Since this stress exceeds 30 % of the allowed dead and live load stresses it will be considered.

Web Members.

$V = -c\left[c_1 e^{cx} - c_2 e^{-cx}\right]$ Johnson, Bryan, Turneaure. Part II.

$c_1 = \dfrac{p}{c^2}\left[\dfrac{1}{e^{cl}+1}\right]$ $c_2 = c_1 e^{cl}$

DESIGN OF LATERAL SYSTEM (cont.)

$$C_1 = \frac{\frac{330}{12}}{.0000000597}\left[\frac{1}{3.162+1}\right] = 110,500,000 \quad C_2 = 3.162 \times$$

$110,500,000 = 349,401,000$

$$V = -.00024\left[110,500,000 - 349,401,000\right] = +57580^{\#}$$

For diagonals area req'd. $= \frac{57580}{18000} = 3.2^{\square''}$

Try 2∠s $4'' \times 4'' \times \frac{1}{2}''$ area $= 3.75^{\square''}$

1 rivet $\frac{.44}{3.31}{}^{\square''}$ O.K.

For laterals use $4'' \times 4'' \times \frac{1}{2}''$ both main and end spans.

Rivets in laterals.

Single shear $= 4418^{\#}$ can be increased 50%

Art. 53 Specifications Stress $= 3.31 \times 18000 = 59500^{\#}$

$\frac{59500}{4418+50\%} =$ 9 rivets.

Redesign of Chords for Truss.

Chord area must be increased on account of wind and temperature stresses. The allowed unit stress can be increased by 30% when wind and temperature stresses are considered. $18000 + 30\% = 23400^{\#\square''}$ will be allowed for the unit stress per sq. in.

DESIGN OF LATERAL SYSTEM (cont.)

Wind stress = 250000# L.L. = 181250#

Temperature stress = 2640#□"

Area for chords = $\frac{250000+181250}{23400-2640}$ = 20.8□"

Try 2∠'s 8"x6"x$\frac{7}{8}$" = 22.96□"
2 rivets = $\underline{1.76}$
21.2□" ∴ O.K.

For end spans – 8"x6"x$\frac{1}{2}$"∠'s will be used.

Rivets in Chord.

Stress = 21.2 x 20760 = 440000#

Bearing on $\frac{7}{8}$" plate at 18000#□" = 11812#

$\frac{440000}{11812}$ = 37 rivets (main span)

End span – S = 12.62 x 12500 = 157800#

$\frac{157800}{6750}$ = 24 rivets

DESIGN OF SUSPENDER RODS.

Where the distance between cables and floorbeams is short steel rods will be used as suspenders. At all other points suspenders consist of high strength steel rope strands.

P = hanger pull per lin. ft. H = horizontal component of cable stress. l = length h = center deflection of cable $s = \frac{h}{l}$

$$P = \frac{8hH}{l^2} \qquad H = \frac{wl}{8s} = \frac{(.48+.19+.21)\,400}{8 \times .095} = 464 \text{ kips.}$$

$$P = \frac{8 \times 38 \times 464}{400^2} = .88 \text{ kips per lin. ft.}$$

P per panel $= 11.1 \times .88 = 9.77$ kips P per hanger $= 4.88$ kips Area req'd. $= \frac{4.88}{16} = .305\,\square''$

Use $\frac{5}{8}''\ \phi$ rods area $= .3068\,\square''$

Longest rod about 46' long. Weight $= 1.043 \times 46 = 48^{\#}$ per panel $= 96^{\#}$ $\frac{96}{11.1} = 8.65^{\#}$ per ft. of truss (max.) Assume $5^{\#}$ per lin. ft. of truss for weight of hangers and details. For cable weight assume $50^{\#}$ per lin. ft.

DESIGN OF CABLES.

Total Load on Cable.

Floor system	=	.232	kips / ft. of cable.
Truss	=	.20	"
Suspender rods	=	.005	"
Live load	=	.48	"
Wind load	=	.33	"
Cable	=	.05	"
	Total	1.297	kips / ft. of cable

T = max. tension in cable. H = horizontal comp. of T. l = span. h = cable sag at center. $s = \frac{h}{l}$

$T = \frac{wl}{8s}\sqrt{1+16s^2} = H\sqrt{1+16s^2}$ Merriman & Jacoby. (Part IV.)

$H = \frac{1.297 \times 400}{8 \times .095} = 683$ kips $T = 683\sqrt{1+16 \times .095^2} =$ $683 \times 1.07 = 730$ kips. Ultimate strength of cable wire = 200,000 #□" Factor of safety = 4 Working strength = 50,000 #□" Area req'd. = $\frac{730000}{50000} = 14.6$ □" Use 7 strands of 39 wires each of No. 2 steel wire. From Roebling's wire catalogue

DESIGN OF CABLES. (cont.)

area of No. 2 wire = .0543 □" $7 \times 39 = 273$ wires

$.0543 \times 273 = 14.8$ □" ∴ O.K.

Cradling of Cable.

b = lateral cradling of cable. h = sag as assumed in a vertical plane. dh = decrease in sag due to cradling. $dh = h - \sqrt{h^2 - b^2} = 38 - \sqrt{38^2 - 4^2} = .21'$

$b = 4'\text{-}0''$ This change in sag increases the cable tension about .1 of 1%. Merriman & Jacoby. (Part IV.)

Cable Deflections. (Loads and temp.)

c = length of cable $c = l\left(1 + \frac{8}{3}s^2 - \frac{32}{5}s^4 + \cdots\right)$

$= 400\left(1 + \frac{8}{3} \times \overline{.095}^2 - \frac{32}{5} \times \overline{.095}^4\right) = 407.392'$ between towers. For temperature – $dh = \frac{3etc}{16s} =$

$$\frac{3 \times .0000072 \times 70 \times 407.392}{16 \times .095} = \pm .405'$$

For live loads – $dh = \frac{3\,dc}{16s}$ $dc = \frac{Tl}{AE}\left(1 - \frac{8}{3}s^2\right)$

T due to live load only. $H = \frac{wl}{8s} = \frac{48 \times 400}{.8 \times .095} =$ 253 kips $T = 1.07H = 1.07 \times 253 = 271$ kips.

DESIGN OF CABLES (cont.)

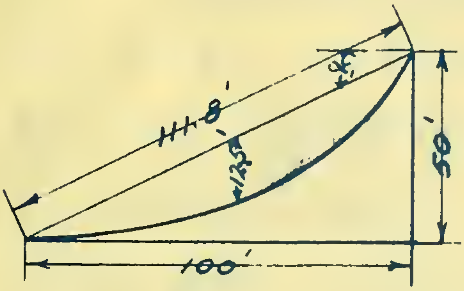
$$dc = \frac{271 \times 400}{14.8 \times 30000}\left(1 - \frac{8}{3} \times \overline{.095}^2\right) = .186' \quad dh = \frac{3 \times .186}{16 \times .095} = .367'$$

Merriman & Jacoby. (Part IV.)

Total deflection = .405 + .367 = .772'

This is provided by cambering trusses. They will be cambered to a radius of 6081' or about 7'-4" in 600'.

DESIGN OF TOWERS.



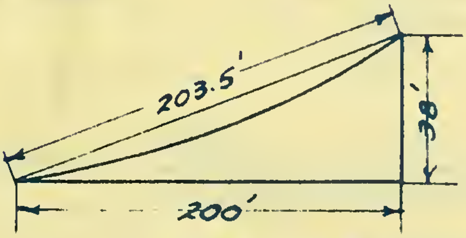
Resultant Loads.

$$H = \frac{w_1 l_1}{8 s_1} = \frac{1.297 \times 100}{8 \times .125} = 129.7 \text{ kips}$$

$$T = H\left[1 + \left(\tan\alpha_1 + \frac{4h_1}{l_1}\right)^2\right]^{\frac{1}{2}}$$

$$= 129.7 \times 1.41 = 183 \text{ kips}$$

$$H = \frac{100}{111.8} \times 183 = 164 \text{ kips} \quad V = \frac{50}{111.8} \times 183 = 82 \text{ kips}$$



$$H = 683 \text{ kips} \qquad T = 730 \text{ kips}$$

$$H = \frac{200}{203.5} \times 730 = 718 \text{ kips}$$

$$V = \frac{38}{203.5} \times 730 = 136 \text{ kips}$$

$$H_R = 718 - 164 = 554 \text{ kips}$$

$$V_R = -82 - 136 = 218 \text{ kips}$$

Bearing Area for Posts.

Allowed bearing on masonry = 400 #□"

Area req'd. = $\frac{218}{.4} = 545$ □" A plate 46" x 41" will be used. Area = 1806 □" ∴ O.K.

Length of Cable.

Main span = 407.392 End spans - $L = l_1\left(1 + \frac{8}{3}s_1^2 + \frac{\tan^2\alpha_1}{2}\right)$

$L = 100\left(1 + \frac{8}{3} \times \overline{.125}^2 + \frac{1}{8}\right) = 116.5$ Total length = 407.39 + 2 x 116.5 = 640.39' Entire length will be about 678'.

DESIGN OF TOWERS

Bending Moment in Towers.

$M = Ql + Vd$ Johnson, Bryan, Turneaure. Part II.

$Q =$ load with V causing a deflection d

$V =$ vertical load $l =$ height of tower

$d = \frac{8}{15} \frac{H h_1^2 l_1}{EI}$ $H =$ for live load (main span.)

$E =$ modulus of elasticity $I =$ moment of truss (end span.) $Q = \frac{Vdc}{\tan cl - cl}$ $c^2 = \frac{V}{EI'}$ I' for tower.

$I = \left(39.82 + 11.5 \times \overline{46.32}^2\right) 2 = 49430$ $H = \frac{wl}{8s} = \frac{.48 \times 400}{8 \times .095} =$ 250 kips

$d = \frac{8 \times 250 \times (12.5 \times 12)^2 \times 100 \times 12}{15 \times 30{,}000 \times 49430} = 2.02''$

$c^2 = \frac{218}{30000 \times 4800} = .00000151$ $c = .00123$

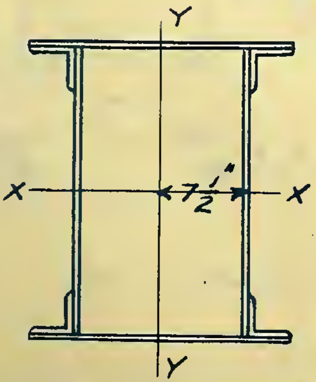
$Q = \frac{218 \times 2.02 \times .00123}{1.428 - .959} = 1.151$ kips

$\tan^{-1} cl = 57.3 \times .00123 \times 65 \times 12 = 55°$ $\tan cl = 1.428$

$cl = .00123 \times 65 \times 12 = .959$

$M = 1.151 \times 65 \times 12 + 218 \times 2.02 = 1340$ kip in.

Design of Post.



Assumed section:

4 L's	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	$= 11.52 \square''$
2 Pls.	$22'' \times \frac{9}{16}''$	$= 24.75 \square''$
2 Pls.	$24'' \times \frac{7}{16}''$	$= 21.00 \square''$
	Total	$= 57.27 \square''$

DESIGN OF TOWERS.

I about x-x axsis — $I_1 = \frac{1}{12} \times 22 \times \overline{\frac{9}{16}}^3 + 24.75 \times \overline{12.28}^2 =$
2910 $\quad I_2 = \frac{1}{12} \times \frac{7}{16} \times \overline{24}^3 = 504 \quad I_3 = 4(3.26 + 2.88 \times \overline{10.96}^2) =$ 1397

Total $= 2910 + 504 + 1397 = 4811''^4$

$r_x \sqrt{\frac{4811}{57.27}} = 9.16''$

I about Y-Y axsis — $I_1 = \frac{1}{12} \times \frac{9}{16} \times \overline{22}^3 = 500$

$I_2 = \frac{1}{12} \times 24 \times \overline{\frac{7}{16}}^3 + 21 \times \overline{7.06}^2 = 1050 \quad I_3 = 4(3.26 + 2.88 \times \overline{8.54}^2)$
$= 853$

Total $= 500 + 1050 + 853 = 2403''^4$

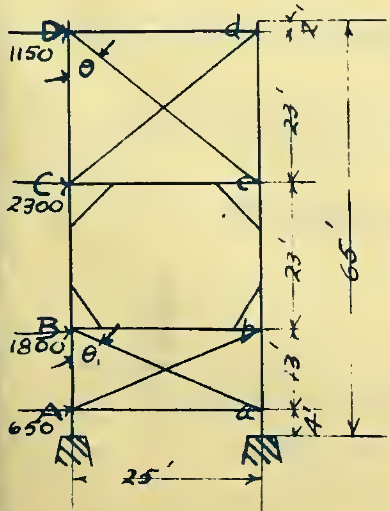
$r_Y = \sqrt{\frac{2403}{57.27}} = 6.48''$

$S = 11000 - \frac{40l}{r} = 11000 - \frac{40 \times 12 \times 65}{6.48} = 6200^{\#\square''}$

$\frac{I}{c} = \frac{M}{S} = \frac{1340}{6.2} = 216$ section modulus required.

$\frac{4811}{12.437} = 386 = \frac{I}{c}$ used. ∴ O.K.

Stresses in Tower Bracing.



Art. 40. Specifications — lateral pressure = 100# per vertical ft.

Dd — 1150 + lateral comp. of cable stress due to cradling =

$1150 + 730 \cos\alpha = 1150 + 14600 = 15750^{\#}$

$Cc = 1150 + 2300 = 3450^{\#}$

$Aa = 1150 + 2300 + 1800 + 650 = 5900^{\#}$

Bb — takes reaction of stiffening trusses.

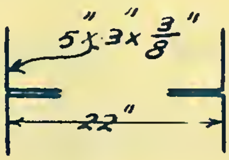
DESIGN OF TOWERS.

For Dd try 4∠'s $3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ with lacing between

∠'s. $S = 16000 - \frac{70\,l}{r}$ $\quad l = 23\frac{1}{2}' = 282''$

$r = 2.26''$ $\quad S = 16000 - \frac{70 \times 282}{2.26} = 7260^{\#\square''}$

Area req'd. $= \frac{11750}{7260} = 1.62^{\square''}$ Area used $= 6.52^{\square''}$ ∴ O.K. For Cc use same section. For Aa 4∠'s $5'' \times 3'' \times \frac{3}{8}''$ ∠'s will be used 22" back to back of ∠'s.



$r = 2.52''$ $\quad S = 16000 - \frac{70 \times 282}{2.52} = 8160^{\#\square''}$

Area req'd. $= \frac{5900}{8160} = .725^{\square''}$ ∴ O.K.

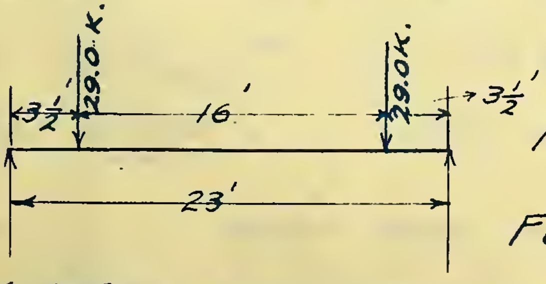
Cd = Dc - Stress $= 1150 \sec\theta = 1150 \times 1.35 = 1550^{\#}$

Ab = aB - Stress $= 5250 \sec\theta_1 = 5250 \times 1.12 = 5880^{\#}$

For these members use 2∠'s $5'' \times 3'' \times \frac{5}{16}''$

Area req'd. $= \frac{5880}{16000} = .367^{\square''}$ ∴ O.K.

Bending mom. on Bb.



$M = \frac{wl^2}{8} = \frac{Rl}{4}$ $\quad R = \frac{4M}{l}$

R = reaction of truss

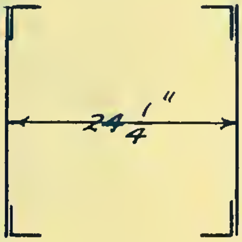
For main span - $R = \frac{4 \times 1450}{400} = 14.5$ K. For end span $R = \frac{725 \times 4}{200} = 14.5$K.

Total $R = 29$ K.

DESIGN OF TOWERS

Mom. = $3.5 \times 29 = 101.5$ kip ft. $= 1{,}220{,}000''^{\#}$

Section modulus reqd. $= \dfrac{1{,}220{,}000}{13{,}000} = 94$ Try 2 pls. $24'' \times \frac{3}{8}''$, 4 ∠s $3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$. Area $= 18 + 6.52 = 24.52^{\square''}$



I about horizontal axsis.

I for pls. $= \frac{1}{12} \times \frac{3}{8} \times \overline{24}^3 \times 2 = 866$

I for ∠'s $= 4(1.42 + 1.63 \times \overline{11.07}^2) = 802$

$\overline{1668}$ Total

Section modulus $= \dfrac{1668}{12} = 139$ ∴ O.K.

Lacing used between ∠'s.

Bracing in BCbc. Compression or tension in brace $= \dfrac{P_1 H_1}{2 l_1}$ Freitag. "Arch. Eng." $= \dfrac{3450 \times 23}{2 \times 4.95} =$ $8000^{\#}$ Use 2 ∠'s $4'' \times 3'' \times \frac{5}{16}''$ area $= 4.18^{\square''}$ $r = 1.27''$.

$S = 16000 - \dfrac{70 \times 4.95 \times 12}{1.27} = 12730^{\#\square''}$ $\dfrac{8000}{12730} = .64^{\square''}$ area reqd. Use this section for all braces.

Rivets – Dd and Cc – $\dfrac{7260 \times 6.52}{4418} = 11$

Aa – $\dfrac{8160 \times 11.44}{4418} = 21$ Ab and Cd $= \dfrac{16000 \times 4.82}{4225} = 18$

Bb $\dfrac{29000}{4418} = 7$ Bracing in BCbc – $\dfrac{16000 \times 4.18}{4225} =$ 16

$4418^{\#}$ = value in single shear.

$4225^{\#}$ = bearing on $\frac{5}{16}''$ pl.

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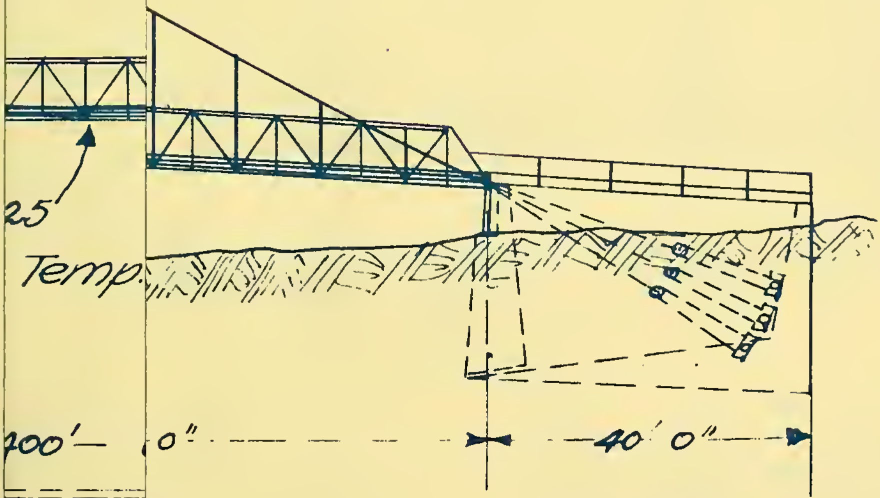
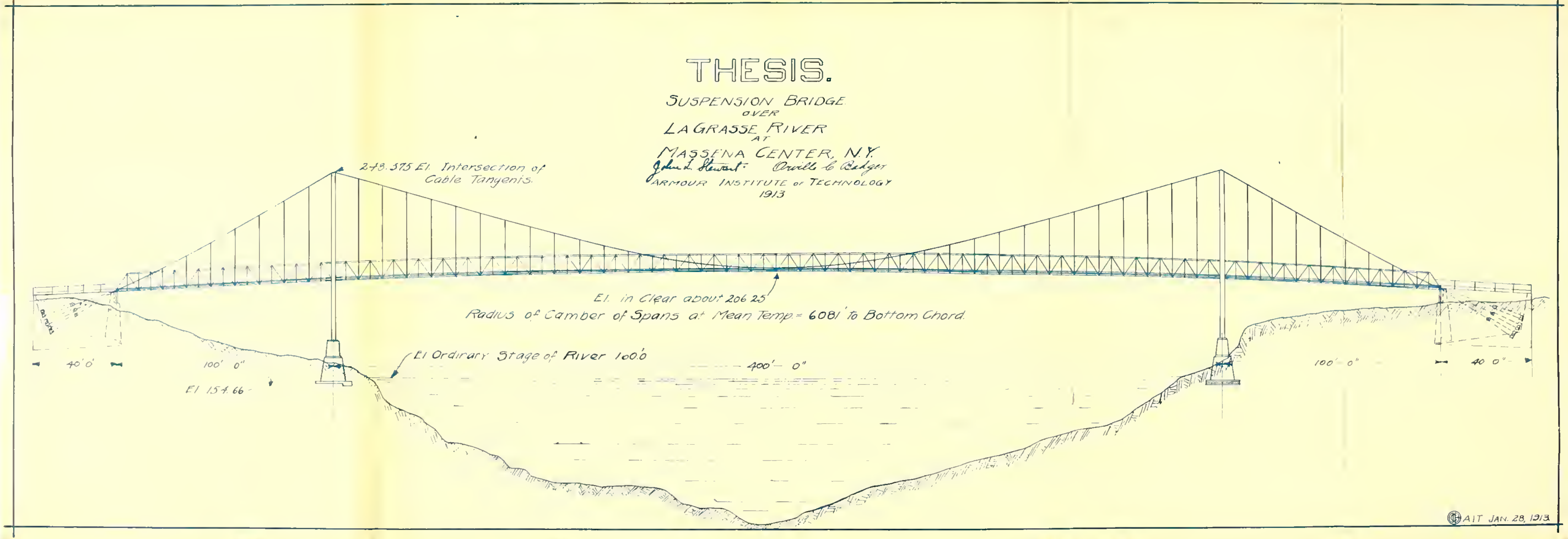
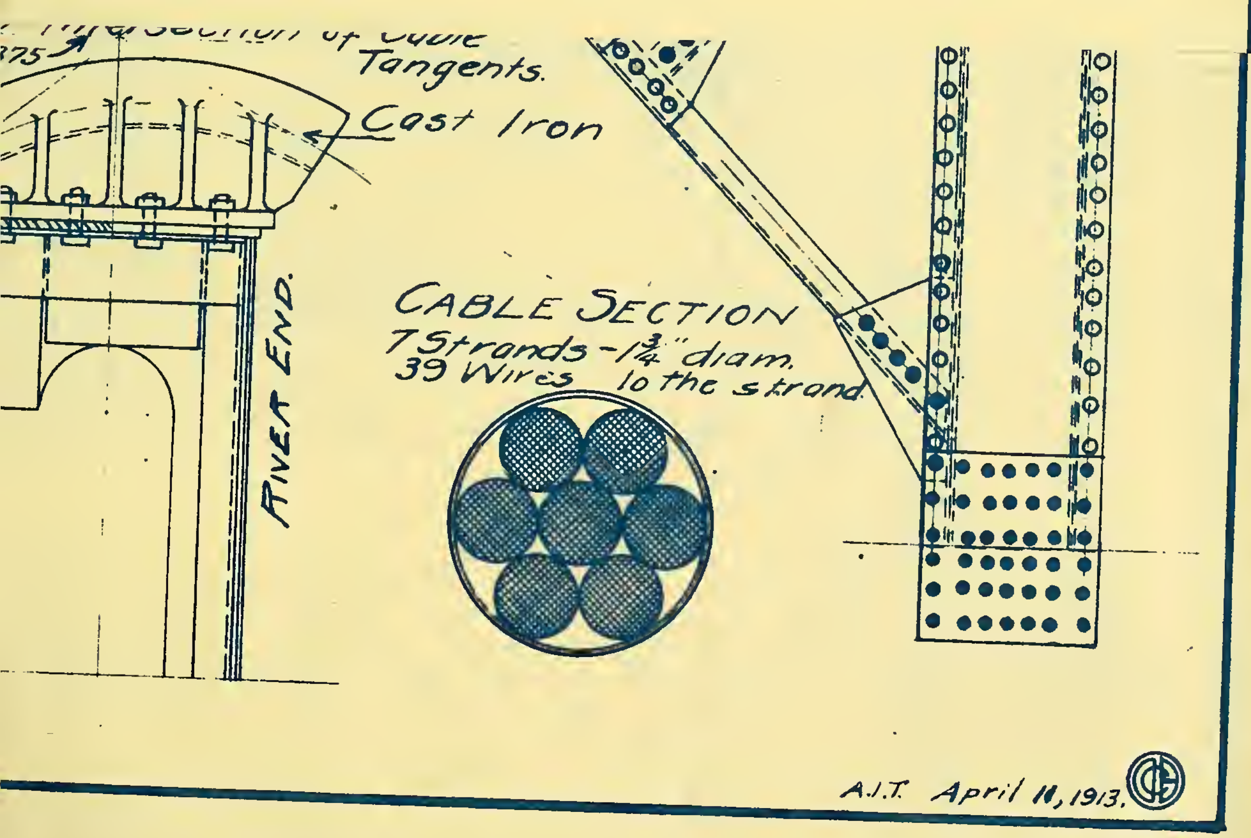
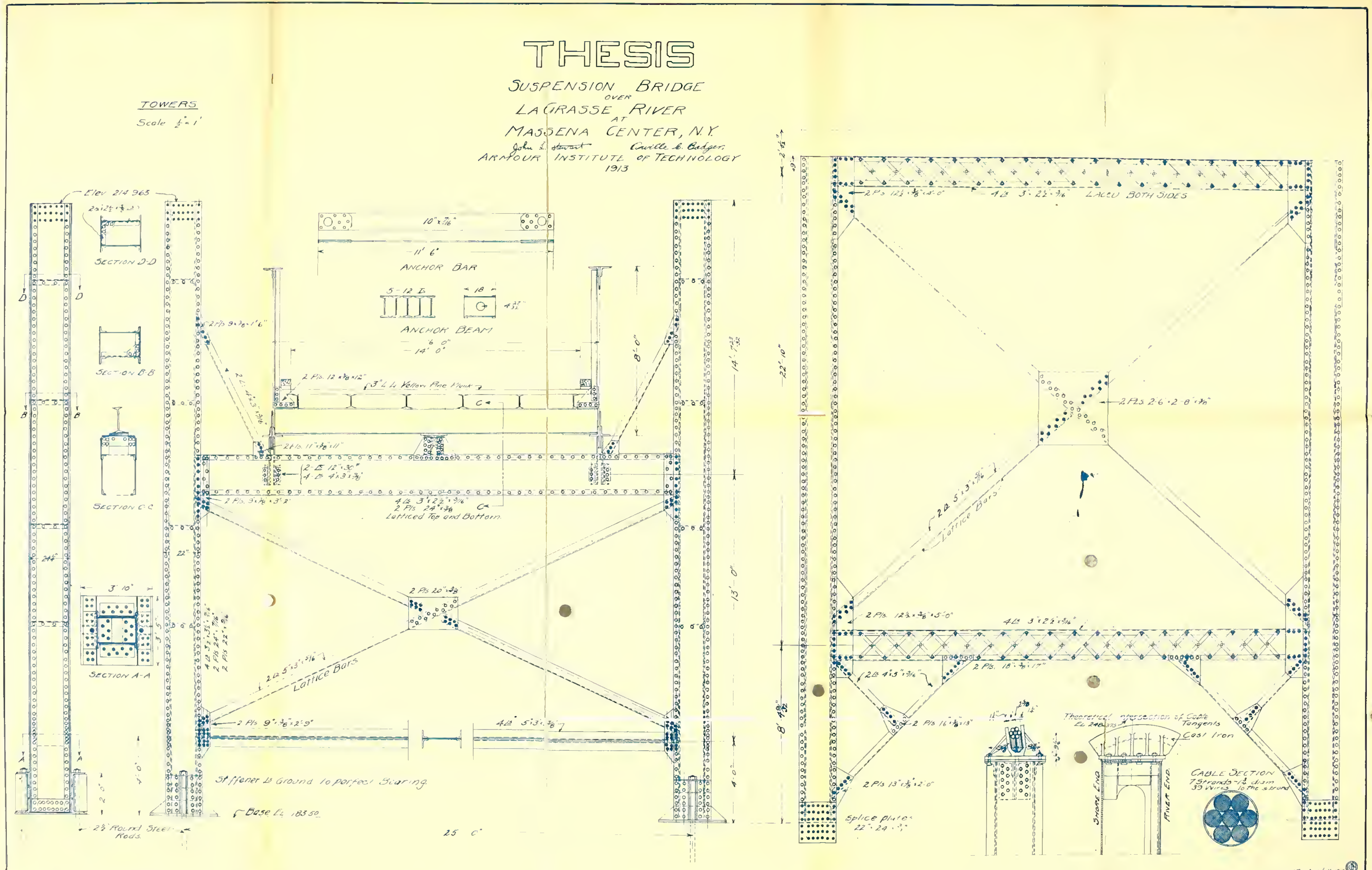
1913.



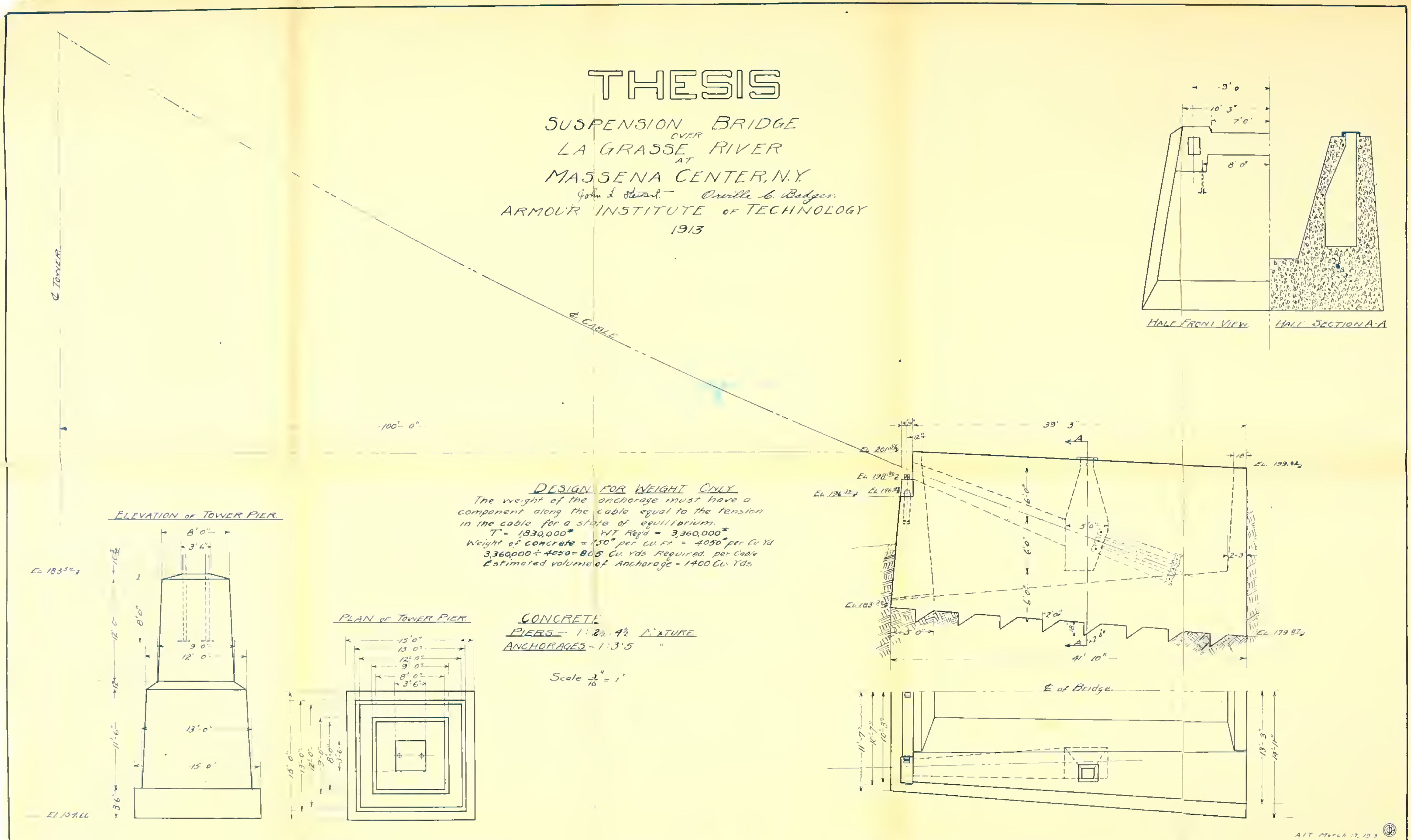
A.I.T. JAN. 28, 1913


THESIS.
SUSPENSION BRIDGE
OVER
LA GRASSE RIVER
AT
MASSENA CENTER, N.Y.
John L. Stewart Orville C. Badger
ARMOUR INSTITUTE OF TECHNOLOGY
1913
248.375 El. Intersection of Cable Tangents.
El. in Clear about 206.25
Radius of Camber of Spans at Mean Temp = 6081' to Bottom Chord.
El Ordinary Stage of River 100.0
40' 0"
100' 0"
400' - 0"
100' 0"
40' 0"
El 154.66
AIT JAN. 28, 1913

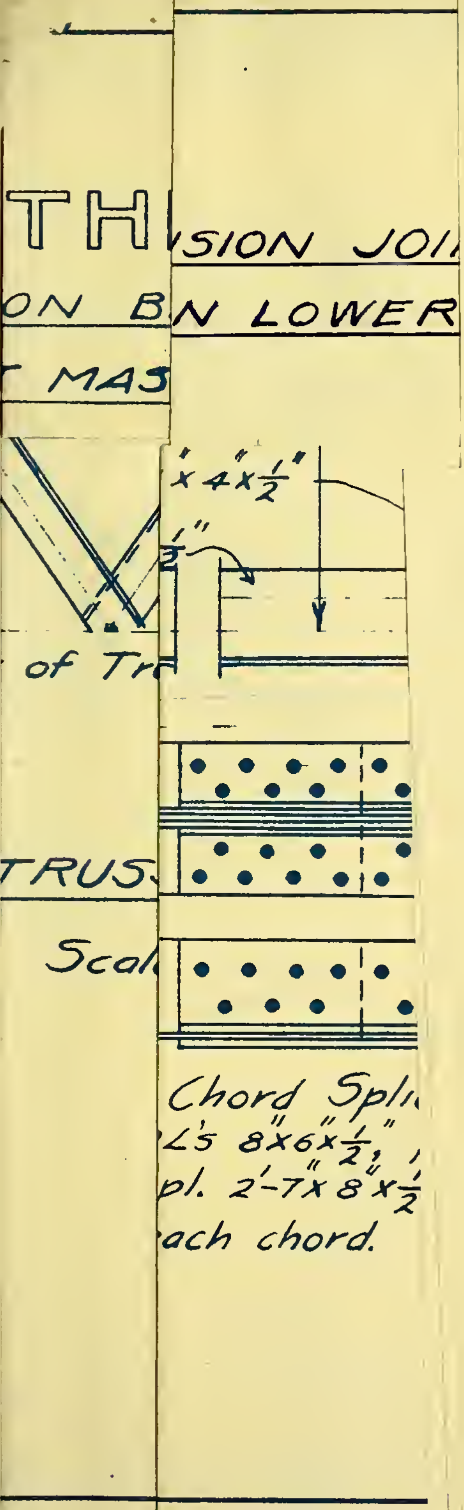

Tangents.
Cast Iron
RIVER END.
CABLE SECTION
7 Strands - 1¾" diam.
39 Wires to the strand.
A.J.T. April 11, 1913.



THESIS
SUSPENSION BRIDGE OVER LA GRASSE RIVER AT MASSENA CENTER, N.Y.
John L. Stewart
Orville L. Badger
ARMOUR INSTITUTE OF TECHNOLOGY
1913
TOWERS
Scale ½" = 1'
Elev. 214.965
SECTION D-D
SECTION B-B
SECTION C-C
SECTION A-A
3' 10"
ANCHOR BAR
10" x 7/16"
11' 6"
5-12 I
18
ANCHOR BEAM
6' 0"
14' 0"
8' 0"
3" L.L. Yellow Pine Plank
2-[12" 30"
4-L 4 x 3 x 3/8
4 L 3' 2½ x 5/16
2 Pls 24" x 3/8
Latticed Top and Bottom
2 Pls 20" x 3/8
2 L 5" x 3" x 5/16
Lattice Bars
4 L 5" x 3" x 3/8
Stiffener Ls Ground to perfect bearing.
Base El. 183.50
2½ Round Steel Rods.
25' 0"
13' 0"
4' 0"
22' 10"
4 L 3" x 2½" x 5/16 LACED BOTH SIDES
2 Pls 2' 6" x 2' 8" x 3/8"
2 Pls 12½ x 3/8 x 5' 0"
2 Pls 18" x 3/8 x 17"
2 Pls 13" x 3/8 x 2' 0"
Splice plates 22" x 24" x 3/4"
Theoretical intersection of Cable Tangents El. 248.325
Cast Iron
SHORE END
RIVER END
CABLE SECTION
7 Strands 1½ diam
39 Wires to the strand
AIT April 11, 1913



THESIS
SUSPENSION BRIDGE
OVER
LA GRASSE RIVER
AT
MASSENA CENTER, N.Y.
John L. Stewart. Orville C. Badger.
ARMOUR INSTITUTE OF TECHNOLOGY
1913
℄ TOWER
℄ CABLE
100'-0"
HALF FRONT VIEW.
HALF SECTION A-A
9'0
10' 3"
7'0"
8'0"
39' 3"
El. 201.3½
El. 198.35
El. 196.30
El. 196.43
El. 199.42
El. 183.30
El. 179.87
5'0"
6'0"
2'0"
41' 10"
2-3
A
A
DESIGN FOR WEIGHT ONLY
The weight of the anchorage must have a component along the cable equal to the tension in the cable for a state of equilibrium.
T = 1830,000# WT Req'd = 3,360,000#
Weight of concrete = 150# per cu. ft = 4050# per Cu Yd
3,360,000 ÷ 4050 = 805 Cu. Yds Required per Cable
Estimated volume of Anchorage = 1400 Cu. Yds
ELEVATION OF TOWER PIER.
El. 183.50
8'0"
3'6"
12'0"
9'0"
13'0"
15'0"
11'-6"
3'6
El. 174.66
PLAN OF TOWER PIER.
15'0"
13'0"
12'0"
9'0"
8'0"
3'6"
CONCRETE
PIERS – 1:2½:4½ MIXTURE
ANCHORAGES – 1:3:5 "
Scale 1/16" = 1'
℄ of Bridge
11'-7"
10'-7"
10'-3"
13'-3"
14'-11"
AIT March 17, 1913


TH
SION JOI
ON B N LOWER
T MAS
x 4"x 1/2"
of Tr
TRUS
Scal
Chord Spli
L's 8"x6"x1/2,"
pl. 2'-7"x 8"x 1/2
ach chord.

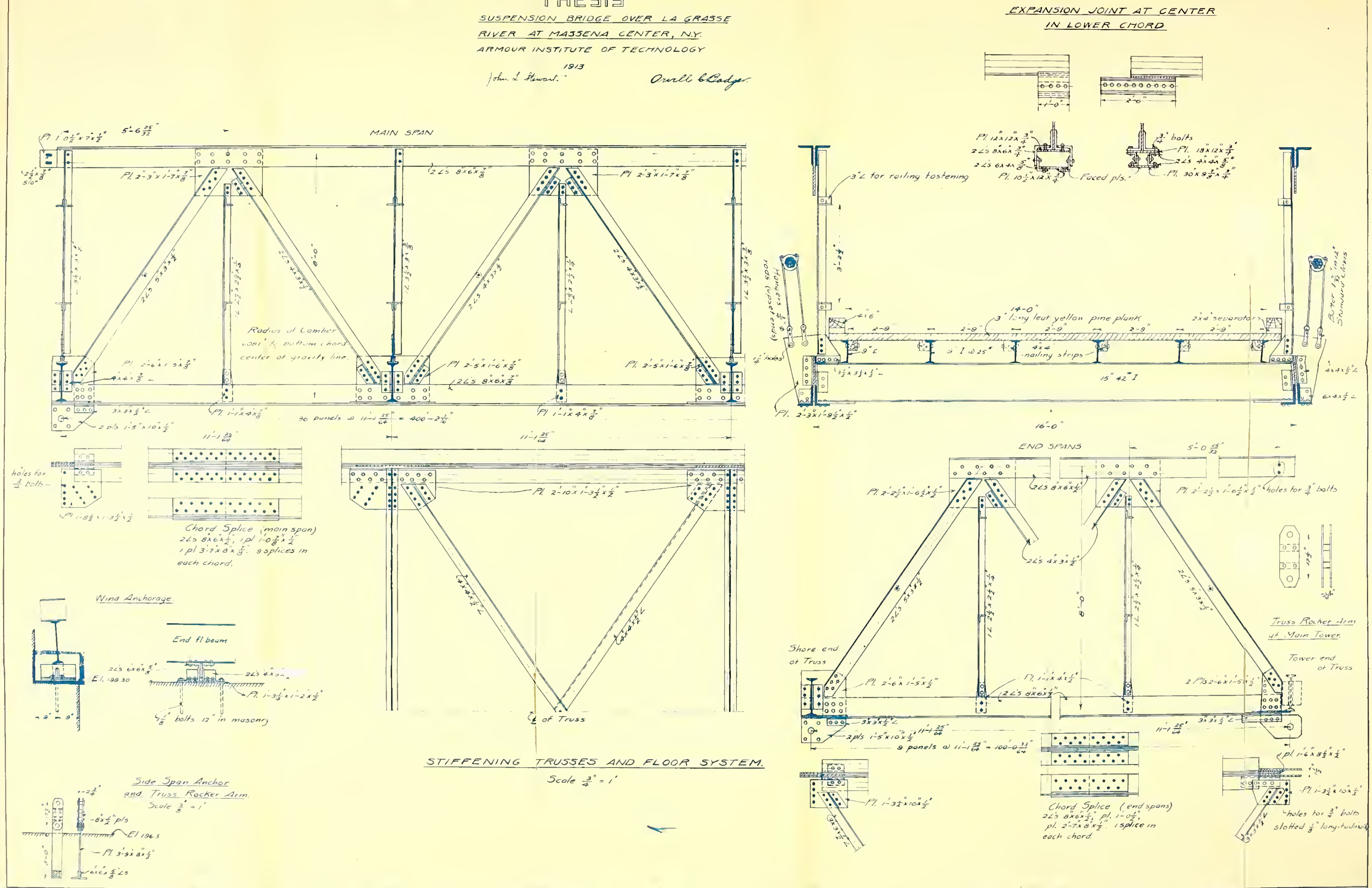
THESIS

SUSPENSION BRIDGE OVER LA GRASSE RIVER AT MASSENA CENTER, N.Y.

ARMOUR INSTITUTE OF TECHNOLOGY

1913

John L. Stewart.   Orrill C. Badger



STIFFENING TRUSSES AND FLOOR SYSTEM.

Scale 3/4" = 1'